APPENDIX VIII

Evaluation of Embankment Stability During Construction

1. Basic Considerations. Embankment stability during construction is affected primarily by pore water pressures induced by the weight of fill placed. When induced pore water pressures are low, stability during construction is generally not a problem. If pore water pressures measured in either the embankment or foundation are high, additional analyses of embankment stability during the construction period should be made, and it may be necessary to: (a) provide berms or flatten slopes, (b) decrease the rate of fill placement, or (c) temporarily discontinue fill operations. Emergency drainage such as electroosmosis may also be considered. The interpretation of measured pore water pressure and evaluation of stability during construction should be regarded as an integral part of embankment design to assure that design expectations and assumptions are consistent with actual embankment and foundation properties.

2. Development of Pore Water Pressure During Construction.

a. General. The development of pore water pressures during construction in either the foundation or in the embankment depends upon the soil properties and the amount of drainage or consolidation occurring during construction. Piezometer observations made during construction should be compared with predicted magnitudes to assess in a general way stability during construction.

b. Embankment Pore Water Pressures. (1) Pore water pressures developed in partially saturated embankment materials during construction depend primarily on (a) fill characteristics such as water content, density, permeability, and compressibility, (b) embankment height, (c) size of core or impervious sections, (d) internal drainage provisions, (e) rate of construction, (f) number of construction seasons, and (g) climatic conditions. Factors involved in pore pressure development in embankments and means for estimating construction pore water pressures are discussed in a recent Corps of Engineers report[15] and are reviewed briefly below.

(2) As additional fill is placed above partially saturated material, the following effects can be observed: (a) the air in the compacted soil is compressed, thereby reducing its volume; (b) the increased pore air pressure causes additional solution of air in the pore water, and an additional volume decrease; (c) pore water pressures are increased; and (d) intergranular stresses are increased by an amount corresponding to the volume decrease caused by compression and solution of air in the soil pore water. Thus, the weight of overlying fill is supported partially by effective stresses in the soil and partially by pore water and air pressures. It is generally assumed, for simplicity and conservatism, that pore air and water pressures are equal, although pore air pressures are actually somewhat higher than pore water pressures. If drainage during construction is ignored, pore air and water pressures estimated[16,17] from application of Boyle's and Henry's laws are conservative. The Brahtz-Hilf procedure for evaluating pore pressures caused by loading a partially saturated soil without drainage taking place is shown in plate VIII-1, together with an example.

(3) When embankments are constructed slowly, in stages, or in two or more construction seasons, significant drainage of pore water may occur and estimated pore pressures may be too high unless consolidation is taken into account. Where stability under stage construction conditions is being investigated, and the gain in shear strength from consolidation occurring between construction periods is taken into account, embankment pore pressures may be estimated from procedures originally developed by the Bureau of Reclamation[17] and extended by Bishop.[18] Dissipation of pore pressures during periods when no fill is placed results in a decrease in soil volume and an increase in effective stress. Bishop pointed out that this increases the stiffness of the soil (i.e. decreases the coefficient of compressibility) and when fill placement is resumed, the induced pore pressures are lower than those that otherwise would have developed. A procedure and an example are shown

in plate VIII-2 for evaluating pore pressures in partially saturated soils with complete dissipation of pore pressures between construction seasons and in plate VIII-3 for partial dissipation of pore pressures in this interval.

(4) The rate of consolidation of partially saturated soils is relatively large during the loading period, when air is compressed and forced into solution, and is relatively slow in later stages when pore pressures decrease and air comes out of solution. The coefficient of consolidation is, therefore, not constant as is often assumed. A "gas factor" to apply to the coefficient of consolidation to account for the change in rate of consolidation of partially saturated soils has been suggested by Gould.[19]

c. Foundation Pore Water Pressures. (1) Excess pore water pressures developed in foundation soils beneath embankments, assuming that significant consolidation does not occur as the fill is placed, can be estimated according to the following equation, developed by Skempton[20]

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]$$

where A and B are experimentally determined pore pressure coefficients, which are illustrated in plate VIII-4 for failure conditions. In general, foundations are assumed saturated and the value of B can be taken as one, so the ratio of induced pore water pressure to the increase in major principal stress becomes

$$\frac{\Delta u}{\Delta\sigma_1} = A + (1 - A)\frac{\Delta\sigma_3}{\Delta\sigma_1}$$

The value of A should correspond to the field value for $\frac{\Delta\sigma_3}{\Delta\sigma_1}$, the ratio of lateral to vertical total stresses, but this is seldom done. The value of $\Delta\sigma_1$ can be taken as approximately equal to the stress imposed by the weight of overlying fill since impervious materials are usually restricted to the central part of embankments where this approximation is reasonably correct. The dependence of excess pore water pressures on the preconsolidation stress of the soil is illustrated by figure 1a, plate VIII-4, and plate VIII-5, assuming a B value of 1.0.

(2) A summary of pore pressures observed in foundations of earth dams is given in a recent Corps of Engineers report.[21] Data presented in it suggest that the approach given above may substantially underestimate pore water pressures developed in shale foundations, but suitable alternative procedures have not been developed. Consequently, recourse must be made to field tests and measurements at sites having such foundation materials. The extent to which this may also be true for hard or highly overconsolidated clays that are not classed as clay shales in unknown.

3. Installation and Uses of Piezometers. a. Piezometers provide the principal means for controlling embankment stability. Undisturbed samples of the soils in which the piezometer tips are installed should be taken large enough in diameter to permit triaxial compression testing of three or four specimens from a common depth. Additional soil samples at other elevations may also be desirable. Piezometer locations and depths should be selected to minimize extrapolation in using the piezometric data in stability analyses.

b. Piezometer observations also may be used to estimate field values for the coefficient of consolidation. These field values may be compared with values assumed in design if consolidation during construction was assumed, and their variation with loading studied as a basis for predicting consolidation under future fill loading. Procedures for estimating the field coefficient of consolidation for one-dimensional compression were developed by Gould[22] and were later extended for combined vertical and radial drainage.[19]

c. Plots of induced pore pressure versus fill load can be used for predicting pore pressure under increased fill heights. However, where soils are partially saturated, the ratio of induced pore pressure to applied load increases as loading continues until all pore air is dissolved; thereafter, the additional pore water pressure approximately equals the added fill weight. Therefore, linear extrapolation of a few early piezometer observations would not account for this nonlinear relation prior to saturation and would be unconservative.

4. Evaluation of Embankment Stability. a. Basic Considerations.

(1) The evaluation of embankment stability during construction should consider all relevant evidence including, in addition to piezometric pressures, such items as (a) movements of settlement plates, (b) horizontal movements of fill and foundation, such as those observed with slope indicators, (c) vertical and horizontal movements of ground at and beyond the embankment toes, (d) vertical and horizontal movements of joints in conduits embedded in the fill, and (e) horizontal and vertical movements of foundations of bridges leading to outlet control towers. Although specific criteria for identifying abnormal behavior cannot be given, repeated observations will show if continuing deformations or anomolous changes in behavior are occurring.

(2) The principal means for assessing embankment stability during construction consist of stability analyses that are directly or indirectly related to pore water pressures. There are various procedures for making such analyses, and it may be desirable to use more than one procedure where embankment stability is questionable. Therefore, several procedures in current use are described in the following paragraphs. All ignore important factors such as nonuniform strain along potential failure surfaces, ultimate strengths that are lower than peak values, redistribution of stresses from embankment loading, and similar aspects that make even the most detailed procedures only approximations to actual conditions.

b. Method A: In Situ Shear Strength Procedure. (1) In this procedure, undisturbed samples are obtained during construction and tested at natural moisture content and density under Q test conditions, without application of back pressure, to determine in situ shear strength. Samples need be obtained only from embankment zones and foundation strata in which high pore pressures have been measured. The shear strength envelope should be determined from the test results in the manner shown in figure 2 of the main text. The undisturbed samples should be obtained at various depths in each soil zone. Each sample should be tested at a single confining pressure of 0.8 times the estimated vertical stress under the in situ condition, since its natural water content and density apply only to the depth at which the sample was obtained.

(2) Stability analyses are then made that are similar to those made in design for the construction condition, except that the shearing resistance along the trial sliding surface is based on the shear resistances determined according to the procedure described above. These analyses consider only the total weight of soil and water in each slice in computing the driving forces and the shear resistance along the sliding surfaces. Water forces on the sides of the slices need not be taken into account since they are internal forces.

(3) The analyses described above apply only to the embankment at the time the undisturbed samples were obtained. If analyses for an increased height of embankment are also desired, additional Q-type tests should be performed in which the confining pressures equal 0.8 times the overburden stresses at the higher fill height. This is conservative since any subsequent consolidation during the fill placement period is ignored.

c. Method B: Measured and Design Pore Pressures. (1) This procedure compares pore water pressures measured during construction with values implicit in the use of Q shear strengths for the construction condition design analyses. If measured pore pressures are less than those implicitly assumed, additional evaluation of embankment stability during construction is not required, unless other field evidence fails to support these observations.

(2) The use of Q-type test results for construction condition design implies that both negative and positive pore water pressures are developed in the embankment and foundation. The pore water pressures inherent in the Q-type laboratory tests can be approximated from Q and S envelopes

and plotted versus total normal stress on the failure plane, as shown in plate VIII-6. If such a plot is prepared, field measured pore water pressures can be simply compared with design expectations, provided piezometers are installed close to the location of the assumed critical failure plane for the design construction condition.

(3) As seen from plate VIII-6, negative pore water pressures must occur in areas of low normal stress if design expectations are to be realized. However, since conventional piezometers are unreliable for measuring negative pore pressures, satisfactory confirmation of design expectations may be impossible to obtain. If high pore pressures are measured in those portions of the embankment or foundation where Q shear strengths are higher than S shear strengths, more detailed methods, such as method A, should be used to check stability during construction.

(4) In lieu of computing pore pressures implied by use of Q test results, they can be measured directly in the laboratory by performing $\overline{Q}$ tests with pore pressure measurements. This requires that the tests be performed slowly so that pore pressures at the center and ends of the test specimen are equalized. Because the test procedures are more complicated and time consuming, $\overline{Q}$ tests for construction condition analyses are not often performed. The same type of porous stone should be used in both the laboratory $\overline{Q}$ tests and the field piezometers so that the pore pressures of low values will have comparable errors.

d. Method C: Modified Swedish or Wedge Method Considering Water Forces. (1) This method is based on procedures described in Appendixes VI and VII. It requires detailed analyses including earth and water forces on the sides and bottom of each slice or wedge segment and should be used only where field and laboratory investigations have been extensive and where embankment soils and foundation materials are not unusual. It should not generally be used for clay shale embankments or foundations.

(2) The water forces on the sides and bottom of each slice or wedge segment can be interpolated from the piezometer observations. For stable

embankments, the soil shearing resistance should be taken as the R strength corresponding to an effective normal stress, prior to start of undrained shear, which is equal to the effective normal stress on the base of each slice or wedge segment, as determined by the stability analysis. When the embankment section is considered to be near failure, the S strength may be used. A near failure condition might be defined by measured horizontal or vertical movements that do not show a decrease with time or by measured pore water pressures that are approaching the stress imposed by the overlying fill. The analysis is similar to that described in Appendixes VI and VII for stability of the downstream slope under a condition of steady seepage.

e. Method D: Modified Bureau of Reclamation Procedure. This procedure consists of comparing field pore water pressures with values predicted according to procedures discussed in paragraph 2 of this appendix and plates VIII-1 to -5. Where this method is used, it should be supplemented by at least one of the other evaluation methods. This method does not consider shear-induced pore pressures.

f. Method E: Modified Swedish or Wedge Method Considering $\bar{\sigma}_1$ and $\bar{\sigma}_3$ Stresses. This method is generally similar to Method C, except that the shear resistance of the soil is the undrained strength corresponding to effective stresses at the start of shear equal to those estimated for field conditions.[23] The following steps are involved:

(1) A plot of shear strength versus effective normal stress on the failure plane at the start of shear is prepared from $\bar{Q}$ or $\bar{R}$ triaxial compression tests. This is done by assuming (after Taylor) that any point in the shearing phase of a $\bar{Q}$ or $\bar{R}$ test corresponds to the start of another test; see plate VIII-7. Next, construct lines of undrained shear strength versus $\bar{\sigma}_{fc}$, the effective normal stress on the failure plane prior to start of undrained shear, for various values of $\bar{\sigma}_1/\bar{\sigma}_3$ as shown in plates VIII-7 and -8.

(2) Assume a trial value of $\left(\bar{\sigma}_1/\bar{\sigma}_3\right)_c$, such as 2, and determine corresponding shear strength parameters c and ϕ from plate VIII-8.

(3) Assume trial safety factors and obtain closure of force polygons for the modified Swedish method, using field measured pore water pressures on the sides and bottom of each slice.

(4) Determine shear stress and corresponding effective normal stress on the base of each slice. Plot as shown in plate VIII-8 to obtain $\bar{\sigma}_1$ and $\bar{\sigma}_3$, and compute $\bar{\sigma}_1/\bar{\sigma}_3$ for each slice.

(5) Compare $\bar{\sigma}_1/\bar{\sigma}_3$ for each slice with value assumed in Step 1. If necessary, revise value of $\left(\bar{\sigma}_1/\bar{\sigma}_3\right)_c$ assumed in Step 1 and repeat Steps 2 through 5.

LEGEND

G = SPECIFIC GRAVITY
n = POROSITY (DECIMAL UNITS)
S = DEGREE OF SATURATION (DECIMAL UNITS)
V = VOLUME
V_a = VOLUME OF AIR
V_s = VOLUME OF SOLIDS
V_v = VOLUME OF VOIDS
V_w = VOLUME OF WATER
w = WATER CONTENT (DECIMAL UNITS)
γ_d = DRY DENSITY
γ_w = UNIT WEIGHT OF WATER

(1) $$S = \frac{w\gamma_d}{\gamma_w\left(1 - \frac{\gamma_d}{G\gamma_w}\right)} = \frac{w}{G\frac{\gamma_w}{\gamma_d} - 1}$$

(2) $$\frac{V_a}{V} = (1 - S)\frac{wG}{S + wG} = 1 - \frac{\gamma_d}{\gamma_w}\left(\frac{1}{G} + w\right)$$

(3) $$n = \frac{V_v}{V} = \frac{\frac{V_a}{V}}{1 - S} = 1 - \frac{\gamma_d}{G\gamma_w} = \frac{wG}{S + wG}$$

(4) $$\frac{V_w}{V} = S\frac{V_v}{V} = \frac{w\gamma_d}{\gamma_w}$$

(5) $$\frac{V_s}{V} = \frac{\gamma_d}{\gamma_w G}$$

(6) $$\frac{V_a}{V} + \frac{V_w}{V} + \frac{V_s}{V} = 1.00$$

GENERAL EQUATIONS RELATING TO FIELD SAMPLE

(7) $$u = \frac{p_a\frac{\Delta H}{H_o}}{\frac{V_a}{V} + h\frac{V_w}{V} - \frac{\Delta H}{H_o}} = \frac{p_a\frac{\Delta H}{H_o}}{n_o\left(1 - S_o + hS_o\right) - \frac{\Delta H}{H_o}}$$

WHERE:

u = AIR AND PORE WATER PRESSURE (ASSUMED EQUAL) INDUCED BY A TOTAL σ_T, EXPRESSED AS GAGE PRESSURE, PSI.

p_a = ABSOLUTE PRESSURE OF AIR IN VOIDS PRIOR TO LOADING BY SUPERPOSED FILL; ASSUMED EQUAL TO ATMOSPHERIC PRESSURE, 14.7 PSI.

$\frac{\Delta H}{H_o}$ = STRAIN OR COMPRESSION OF SOIL PER UNIT VOLUME (ONE-DIMENSIONAL COMPRESSION) CAUSED BY A TOTAL LOAD σ_T

h = HENRY'S CONSTANT OF SOLUBILITY OF AIR IN WATER BY VOLUME (0.0198 AT 68°F)

n_o = INITIAL POROSITY, PRIOR TO LOADING

S_o = INITIAL DEGREE OF SATURATION, PRIOR TO LOADING

EQUATION 7 IS VALID ONLY FOR PARTIALLY SATURATED SOILS AND AT POINT WHEN SOIL BECOMES SATURATED; I.E. WHEN

$$\frac{\Delta H}{H_o} = \frac{V_a}{V}$$

THE INDUCED AIR AND PORE WATER PRESSURE WHEN SOIL BECOMES SATURATED IS

(8) $$u_s = \frac{p_a V_a/V}{h V_w/V} = \frac{p_a}{h} \times \frac{1 - S_o}{S_o} = 50.5\ p_a\frac{1 - S_o}{S_o}$$

u_s = GAGE PRESSURE OF AIR AND WATER IN VOIDS OF SOIL

EXAMPLE

INITIAL CONDITION OF SOIL BEFORE LOADING

w = 17.1% = 0.171
γ_d = 109.2 PCF
G = 2.68

STEP 1:

EQ 1 $$S_o = \frac{wG}{G \times \frac{\gamma_w}{\gamma_d} - 1} = \frac{0.171 \times 2.68}{2.68 \times \frac{62.4}{109.2} - 1} = 0.864$$

EQ 2 $$\frac{V_a}{V} = 1 - \frac{\gamma_d}{\gamma_w}\left(\frac{1}{G} + w\right) = 1 - \frac{109.2}{62.4}\left(\frac{1}{2.68} + 0.171\right) = 0.048$$

EQ 3 $$n = \frac{V_v}{V} = 1 - \frac{109.2}{62.4 \times 2.68} = 1 - 0.653 = 0.347$$

EQ 4 $$\frac{V_w}{V} = 0.171 \times \frac{109.2}{62.4} = 0.299$$

EQ 5 $$\frac{V_s}{V} = \frac{109.2}{62.4 \times 2.68} = 0.653$$

EQ 6 $$\frac{V_a}{V} + \frac{V_w}{V} + \frac{V_s}{V} = 0.048 + 0.299 + 0.653 = 1.00 \quad \text{(CHECKS)}$$

STEP 2: CONSOLIDATION TEST RESULTS ARE PLOTTED IN FIG. 1 AS $\bar{\sigma}$ VS $\frac{\Delta H}{H_o}$

STEP 3: EQ 8 $$u_s = 50.5\ p_a\frac{1 - S_o}{S_o} = 50.5 \times 14.7 \times \frac{1 - 0.864}{0.864} = 116.6 \text{ PSI}$$

STEPS 4, 5, AND 6

EQ 7 $$u = \frac{p_a\frac{\Delta H}{H_o}}{n_o\left(1 - 0.98\ S_o\right) - \frac{\Delta H}{H_o}} = \frac{14.7 \times \frac{\Delta H}{H_o}}{0.347\ (1 - 0.98 \times 0.864) - \frac{\Delta H}{H_o}}$$

$$u = \frac{14.7 \times \frac{\Delta H}{H_o}}{0.0531 - \frac{\Delta H}{H_o}}$$

$\frac{\Delta H}{H_o}$	$\bar{\sigma}$ PSI	$0.0531 - \frac{\Delta H}{H_o}$	u PSI	$\sigma_T = \bar{\sigma} + u$
0.005	4.5	0.04	1.5	6.0
0.010	9.0	0.04	3.4	12.4
0.015	14.5	0.03	5.8	20.3
0.020	20.5	0.03	.9	29.4
0.024	25.0	0.02	12.1	37.1
0.030	33.5	0.02	19.1	52.6
0.035	41.0	0.01	28.4	69.4
0.041	50.0	0.01	49.8	99.8
0.048*	57.0	0.005	138.4	195.4

*SATURATION, $\frac{\Delta H}{H_o} = \frac{V_a}{V}$

STEP 7: PLOT u VS $\frac{\Delta H}{H_o}$ AND σ_T VS $\frac{\Delta H}{H_o}$ IN FIG. 1 AND σ_T VS u IN FIG. 2.

PROCEDURE FOR COMPUTING PORE PRESSURES

1. COMPUTE EQUATIONS 1-6 AND CHECK RESULTS USING EQUATION 6.
2. PLOT CONSOLIDATION TEST RESULTS AS $\frac{\Delta H}{H_o}$ VS $\bar{\sigma}$ (PLOT REQUIRED ONLY FOR VALUES OF $\frac{\Delta H}{H_o}$ FROM 0 TO $\frac{\Delta H}{H_o} = \frac{V_a}{V}$).
3. COMPUTE INDUCED PORE AND AIR PRESSURE CORRESPONDING TO SATURATION FROM EQUATION 8.
4. COMPUTE INDUCED PORE PRESSURE FROM EQUATION 7 FOR VARIOUS VALUES OF $\frac{\Delta H}{H_o}$ EQUAL TO AND LESS THAN $\frac{\Delta H}{H_o} = \frac{V_a}{V}$
5. OBTAIN $\bar{\sigma}$ CORRESPONDING TO VALUES OF $\frac{\Delta H}{H_o}$ USED IN STEP 4 FROM PLOT PREPARED IN STEP 2.
6. COMPUTE σ_T VALUES FOR EACH VALUE OF $\frac{\Delta H}{H_o}$; $\sigma_T = \bar{\sigma} + u$.
7. PLOT u VS σ_T; ALSO PLOT u VS $\frac{\Delta H}{H_o}$ AND $\bar{\sigma}$ VS $\frac{\Delta H}{H_o}$ AND σ_T VS $\frac{\Delta H}{H_o}$.

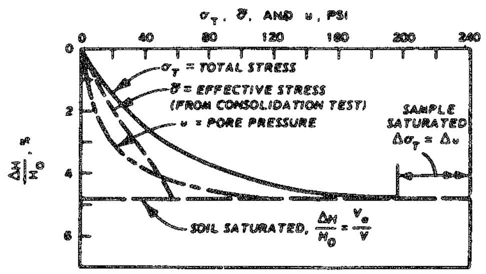


FIG. 1. TOTAL AND EFFECTIVE STRESSES AND PORE PRESSURE VS DEFORMATION

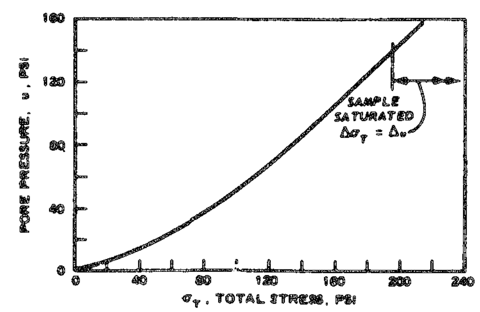


FIG. 2. TOTAL STRESS VS PORE PRESSURE

PORE PRESSURES IN PARTIALLY SATURATED SOILS

NO DRAINAGE DURING LOADING

Plate VIII-1

PORE PRESSURES INDUCED BY ADDITIONAL LOADING WITHOUT DRAINAGE, FOLLOWING AN INTERVAL IN WHICH NO FILL WAS PLACED AND DURING WHICH COMPLETE DISSIPATION OF PORE PRESSURES OCCURRED

AT THE END OF FIRST CONSTRUCTION SEASON

TOTAL FILL LOAD $= \sigma_{T_1}$

PORE PRESSURE $= u_1$

EFFECTIVE STRESS $= \bar{\sigma}_1$

VOLUME DECREASE $= \frac{\Delta H_1}{H_e}$

AT START OF SECOND CONSTRUCTION SEASON

TOTAL FILL LOAD $= \sigma_{T_1}$

PORE PRESSURE $= u_2 = 0$

EFFECTIVE STRESS $= \bar{\sigma}_{T_1} = \sigma_{T_1}$

VOLUME DECREASE $= \frac{\Delta H_2}{H_e}$

AT THE START OF SECOND CONSTRUCTION SEASON, POROSITY $= n_2$

$$n_2 = n_e - \frac{\Delta H_2}{H_e} \tag{9}$$

DURING SECOND CONSTRUCTION SEASON, INDUCED PORE PRESSURE IS GIVEN BY

$$u = \frac{p_e \frac{\Delta H'}{H_e}}{n_2 (1 - S_e + hS_e) - \frac{\Delta H'}{H_e}} \tag{10}$$

WHERE

$\frac{\Delta H'}{H_e}$ = ADDITIONAL VOLUME CHANGE, MEASURED FROM $\frac{\Delta H_2}{H_e}$

S_e = DEGREE OF SATURATION PRIOR TO LOADING (AT START OF SECOND CONSTRUCTION SEASON, ASSUMED EQUAL TO S_e AT START OF FIRST CONSTRUCTION SEASON).

NOTE: SEE PLATE VIII-1 FOR BASIC DEFINITION OF TERMS.

EXAMPLE

ASSUMPTIONS: AT END OF FIRST CONSTRUCTION SEASON

TOTAL FILL LOAD $= \sigma_{T_1} = 37.8$ PSI

PORE PRESSURE $= u = 12.8$ PSI

EFFECTIVE STRESS $= \bar{\sigma}_1 = 25.0$ PSI

VOLUME DECREASE $= \frac{\Delta H_1}{H_e} = 0.024$ OR 2.4%

ASSUME THAT PORE PRESSURE DISSIPATES 100% DURING INTERVAL BETWEEN FIRST AND SECOND CONSTRUCTION SEASONS.

AT START OF SECOND CONSTRUCTION SEASON

PORE PRESSURE $= u_2 = 0$

EFFECTIVE STRESS $= \bar{\sigma}_{T_1} = 37.8$ PSI

VOLUME DECREASE $= \frac{\Delta H_2}{H_e} = 0.033$ OR 3.3%

COMPUTATIONS: AT START OF SECOND CONSTRUCTION SEASON

EQ 9 POROSITY $= n_2 = n_e - \frac{\Delta H_2}{H_e} = 0.345 - 0.033 = 0.312$

DURING SECOND CONSTRUCTION SEASON

$$\text{EQ 10} \quad u = \frac{p_e \frac{\Delta H'}{H_e}}{n_2 (1 - 0.98 S_e) - \frac{\Delta H'}{H_e}} = \frac{14.7 \times \frac{\Delta H'}{H_e}}{0.312 (1 - 0.98 \times 0.864) - \frac{\Delta H'}{H_e}}$$

$$u = \frac{14.7 \times \frac{\Delta H'}{H_e}}{0.0477 - \frac{\Delta H'}{H_e}}$$

$\frac{\Delta H'}{H_e}$	$\frac{\Delta H}{H_e}$	$0.0477 - \frac{\Delta H'}{H_e}$	u PSI	$\bar{\sigma}$ PSI	$\sigma_T = \bar{\sigma} + u$ PSI
0	0.033	0.0477	0	37.8	37.8
0.002	0.035	0.0457	0.6	41.0	41.6
0.008	0.041	0.0397	3.0	50.0	53.0
0.0126	0.0456	0.0351	5.3	57.0	62.3

PLOT $\bar{\sigma}$, u, AND σ_T VS $\frac{\Delta H}{H_e}$ IN FIG. 1.

PLOT σ_T VS u IN FIG. 2.

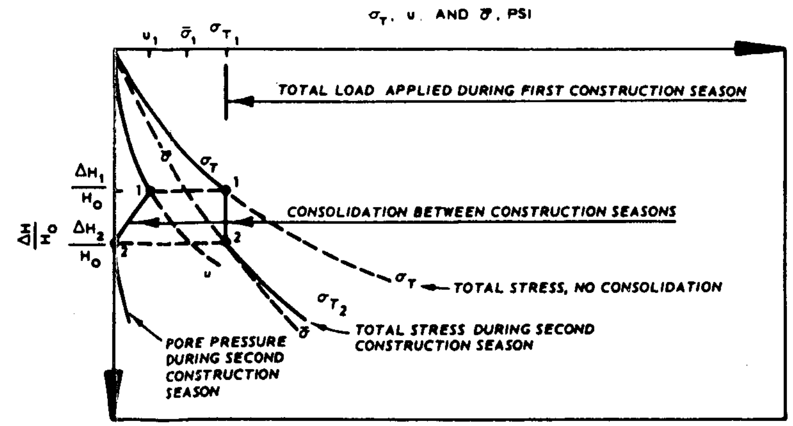


FIG. 1. TOTAL AND EFFECTIVE STRESSES AND PORE PRESSURE VS DEFORMATION

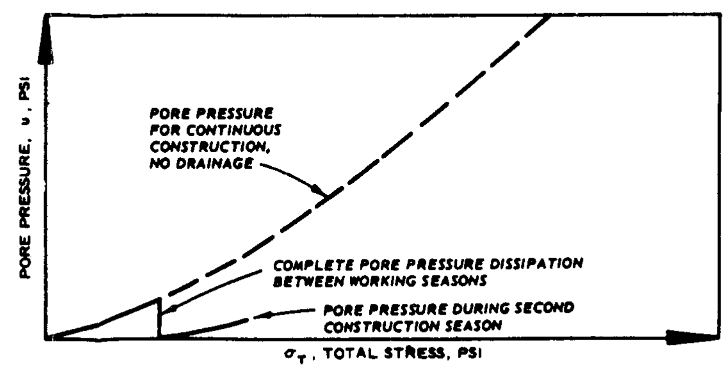


FIG. 2. TOTAL STRESS VS PORE PRESSURE

PORE PRESSURES IN PARTIALLY SATURATED SOILS

EFFECT OF COMPLETE DISSIPATION OF PORE PRESSURE BETWEEN CONSTRUCTION SEASONS

PORE PRESSURES INDUCED BY ADDITIONAL LOADING AFTER AN INTERVAL IN WHICH NO FILL WAS PLACED AND DURING WHICH PARTIAL DISSIPATION OF PORE PRESSURE OCCURRED

AT THE END OF FIRST CONSTRUCTION SEASON

TOTAL FILL LOAD $= \sigma_{T_1}$

PORE PRESSURE $= u_1$

EFFECTIVE STRESS $= \bar{\sigma}_1$

VOLUME DECREASE (STRAIN) $= \dfrac{\Delta H_1}{H_o}$

AT START OF SECOND CONSTRUCTION SEASON

TOTAL FILL LOAD $= \sigma_{T_1}$

PORE PRESSURE $= u_2$

EFFECTIVE STRESS $= \bar{\sigma}_2$

VOLUME DECREASE $= \dfrac{\Delta H_2}{H_o}$

AT START OF SECOND CONSTRUCTION SEASON

$$\text{(9)} \quad \text{POROSITY} = n_2 = n_o - \frac{\Delta H_2}{H_o}$$

$$\text{(11)} \quad \text{DEGREE OF SATURATION} = S_2 = \frac{S_o\left(1 + \dfrac{u_2}{p_a}\right)}{1 + \dfrac{u_2}{p_a}(1 - h)\,S_o}$$

DURING SECOND CONSTRUCTION SEASON, INDUCED PORE PRESSURE IS GIVEN BY

$$\text{(12)} \quad \Delta u = \frac{(p_a + u_2)\dfrac{\Delta H'}{H_o}}{n_2(1 - S_2 + hS_2) - \dfrac{\Delta H'}{H_o}}$$

WHERE Δu = ADDITIONAL PORE PRESSURE CAUSED BY LOADING DURING SECOND CONSTRUCTION SEASON.

p_a = ATMOSPHERIC PRESSURE, 14.7 PSI

u_2 = PORE PRESSURE AT START OF SECOND CONSTRUCTION SEASON.

$\dfrac{\Delta H'}{H_o}$ = ADDITIONAL VOLUME CHANGE, MEASURED FROM $\dfrac{\Delta H_2}{H_o}$

n_2 = POROSITY AT START OF SECOND CONSTRUCTION SEASON

S_2 = DEGREE OF SATURATION AT START OF SECOND CONSTRUCTION SEASON.

NOTE: SEE PLATE VIII-1 FOR BASIC DEFINITION OF TERMS.

ASSUME: AT END OF FIRST CONSTRUCTION SEASON

$\sigma_{T_1} = 37.8$ PSI

$u = 12.8$ PSI

$\bar{\sigma} = 25.0$ PSI

$\dfrac{\Delta H}{H_o} = 0.025$ OR 2.4%

ASSUME THAT PORE PRESSURE DISSIPATES 50% DURING INTERVAL BETWEEN FIRST AND SECOND CONSTRUCTION SEASONS

AT START OF SECOND CONSTRUCTION SEASON

$u_2 = 6.4$ PSI

$\bar{\sigma} = 31.4$ PSI

$\dfrac{\Delta H_2}{H_o} = 0.0285$ OR 2.85%

COMPUTATIONS: AT START OF SECOND CONSTRUCTION SEASON

$$\text{(9)} \quad n_2 = n_o - \frac{\Delta H_2}{H_o} = 0.345 - 0.0285 = 0.3165$$

$$\text{(11)} \quad S_2 = \frac{S_o\left(1 + \dfrac{u_2}{p_a}\right)}{1 + \dfrac{u_2}{p_a}(1 - h)\,S_o} = \frac{0.864\left(1 + \dfrac{6.4}{14.7}\right)}{1 + \dfrac{6.4}{14.7} \times 0.98 \times 0.864} = 0.906$$

DURING SECOND CONSTRUCTION SEASON

$$\text{(12)} \quad \Delta u = \frac{(p_a + u_2)\dfrac{\Delta H'}{H_o}}{n_2(1 - 0.98S_2) - \dfrac{\Delta H'}{H_o}} = \frac{(14.7 + 6.4)\dfrac{\Delta H'}{H_o}}{0.3165(1 - 0.98 \times 0.906) - \dfrac{\Delta H'}{H_o}}$$

$$\Delta u = \frac{21.1 \times \dfrac{\Delta H'}{H_o}}{0.0345 - \dfrac{\Delta H'}{H_o}}$$

$\frac{\Delta H'}{H_o}$	$\frac{\Delta H}{H_o}$	$0.0348 - \frac{\Delta H'}{H_o}$	Δu PSI	$u = \Delta u + u_2$ PSI	$\bar{\sigma}$ PSI	$\sigma_T = \bar{\sigma} + u$ PSI
0	0.0285	0.0345	0	6.4	31.4	37.8
0.0015	0.030	0.0330	1.0	7.4	33.5	40.9
0.0065	0.035	0.0280	4.9	11.3	41.0	52.3
0.0125	0.041	0.0220	12.0	18.4	50.0	68.4
0.0171	0.0456	0.0174	20.7	27.1	57.0	84.1

PLOT $\bar{\sigma}$, u, AND σ_T VS $\frac{\Delta H}{H_o}$ IN FIG. 1.

PLOT σ_T VS u IN FIG. 2.

EXAMPLE

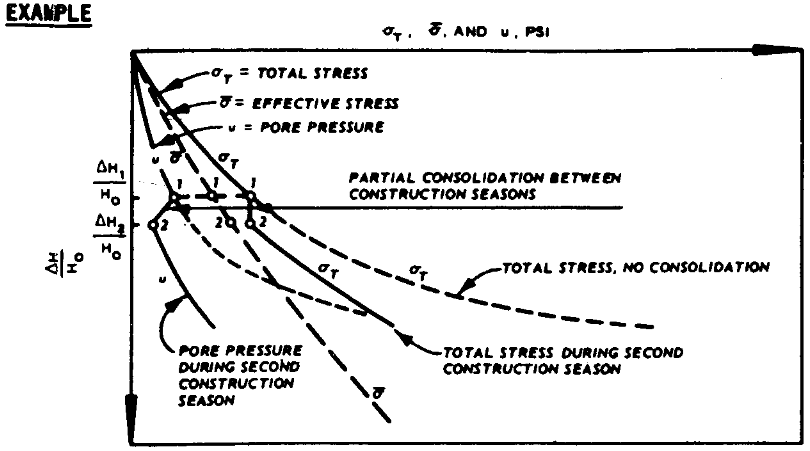


FIG. 1. TOTAL AND EFFECTIVE STRESSES AND PORE PRESSURE VS DEFORMATION

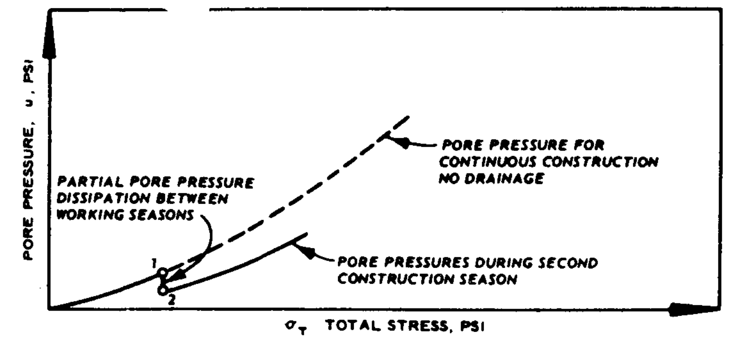
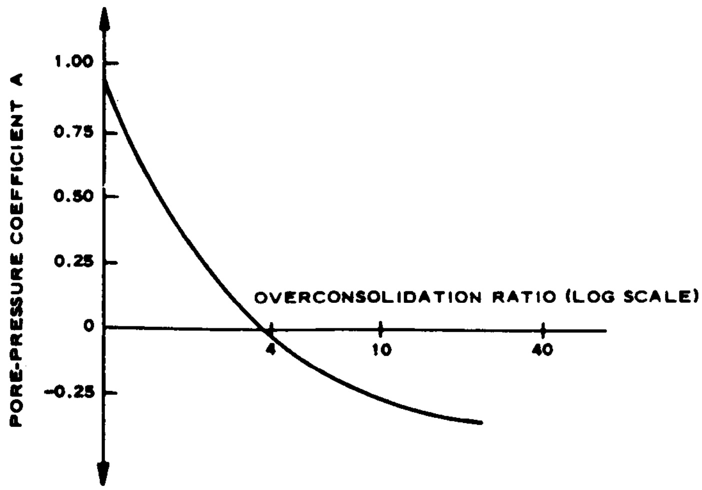


FIG. 2. TOTAL STRESS VS PORE PRESSURE

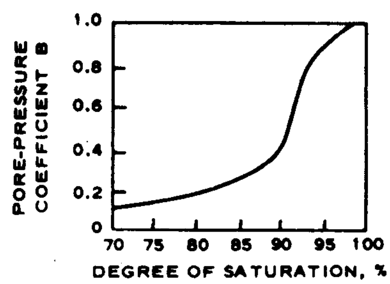
PORE PRESSURES IN PARTIALLY SATURATED SOILS

EFFECT OF PARTIAL DISSIPATION OF PORE PRESSURE BETWEEN CONSTRUCTION SEASONS

Plate VIII-3



a. PORE-PRESSURE COEFFICIENT A VERSUS OVERCONSOLIDATION RATIO; COEFFICIENT MEASURED AT FAILURE, STRESS INCREASING



b. PORE-PRESSURE COEFFICIENT B VERSUS DEGREE OF SATURATION; COEFFICIENT MEASURED AT FAILURE, STRESS INCREASING. CURVE APPLIES TO ONE SOIL ONLY, UNDER PARTICULAR CONDITIONS OF TEST

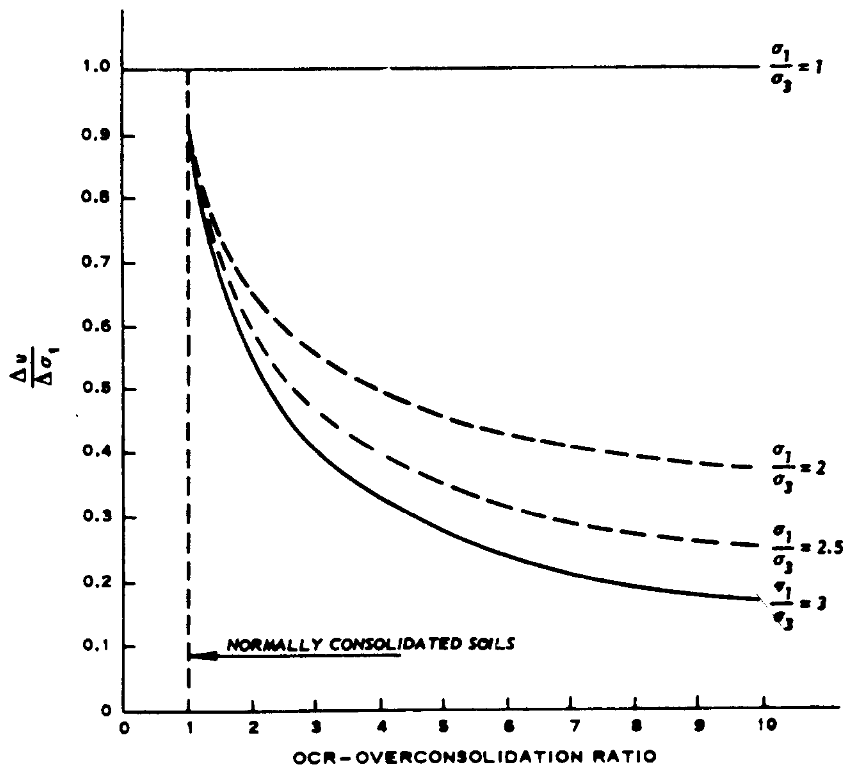
PORE PRESSURE COEFFICIENTS A AND B

Plate VIII-4

$$\Delta u = B\,[\Delta\sigma_3 + A\,(\Delta\sigma_1 - \Delta\sigma_3)]$$ (PER SKEMPTON)

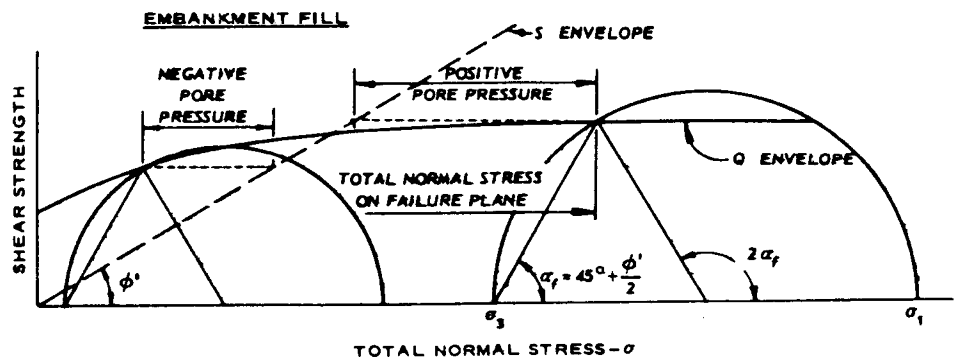
WHERE Δu = INCREASED PORE WATER PRESSURE
$\Delta\sigma_3$ = INCREASE IN MINOR PRINCIPAL STRESS
$\Delta\sigma_1$ = INCREASE IN MAJOR PRINCIPAL STRESS
B = 1.0 FOR SATURATED SOILS
A = FACTOR DEPENDENT ON OVERCONSOLIDATION RATIO

THEN $$\frac{\Delta u}{\Delta\sigma_1} = A + (1 - A)\frac{\Delta\sigma_3}{\Delta\sigma_1}$$

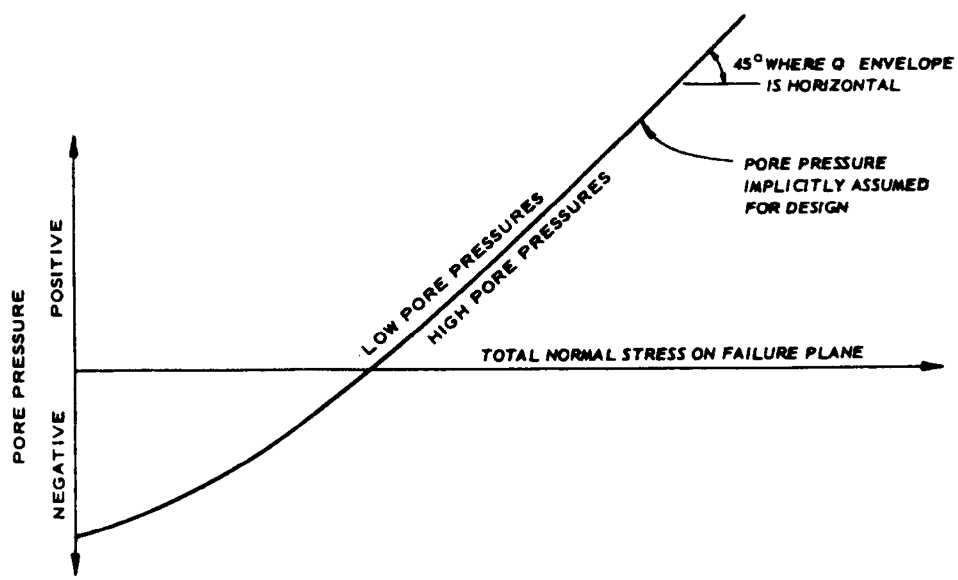


DEVELOPMENT OF EXCESS PORE WATER PRESSURES

Plate VIII-5



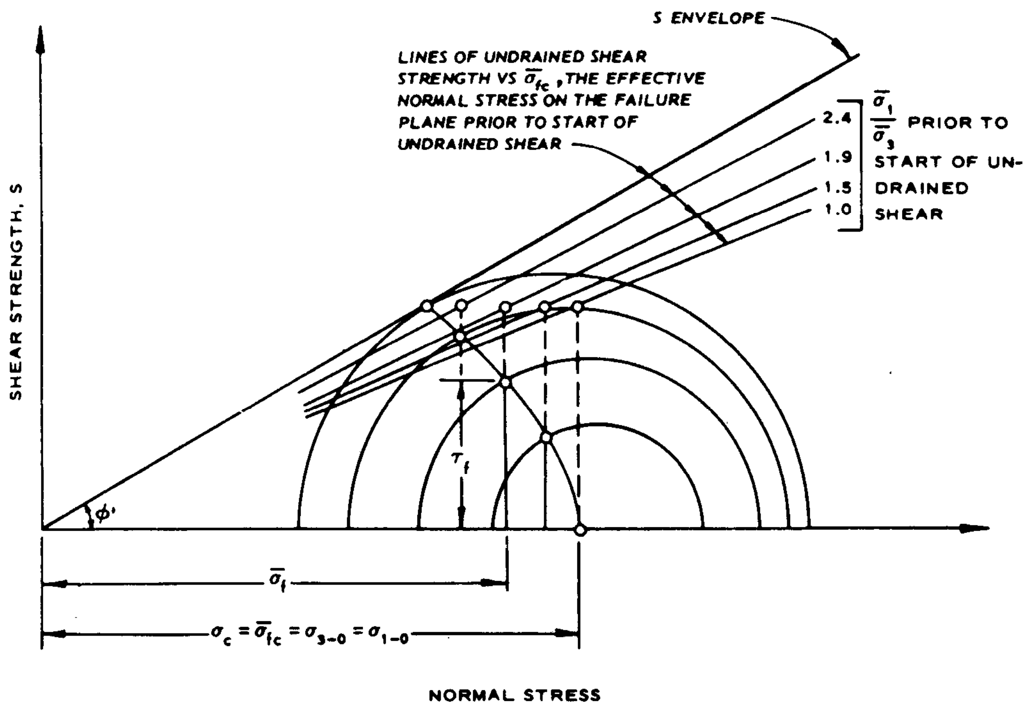
PORE PRESSURES IMPLIED WHEN Q SHEAR STRENGTHS ARE USED FOR CONSTRUCTION CONDITION DESIGN



PORE PRESSURE VS TOTAL NORMAL STRESS ON FAILURE PLANE

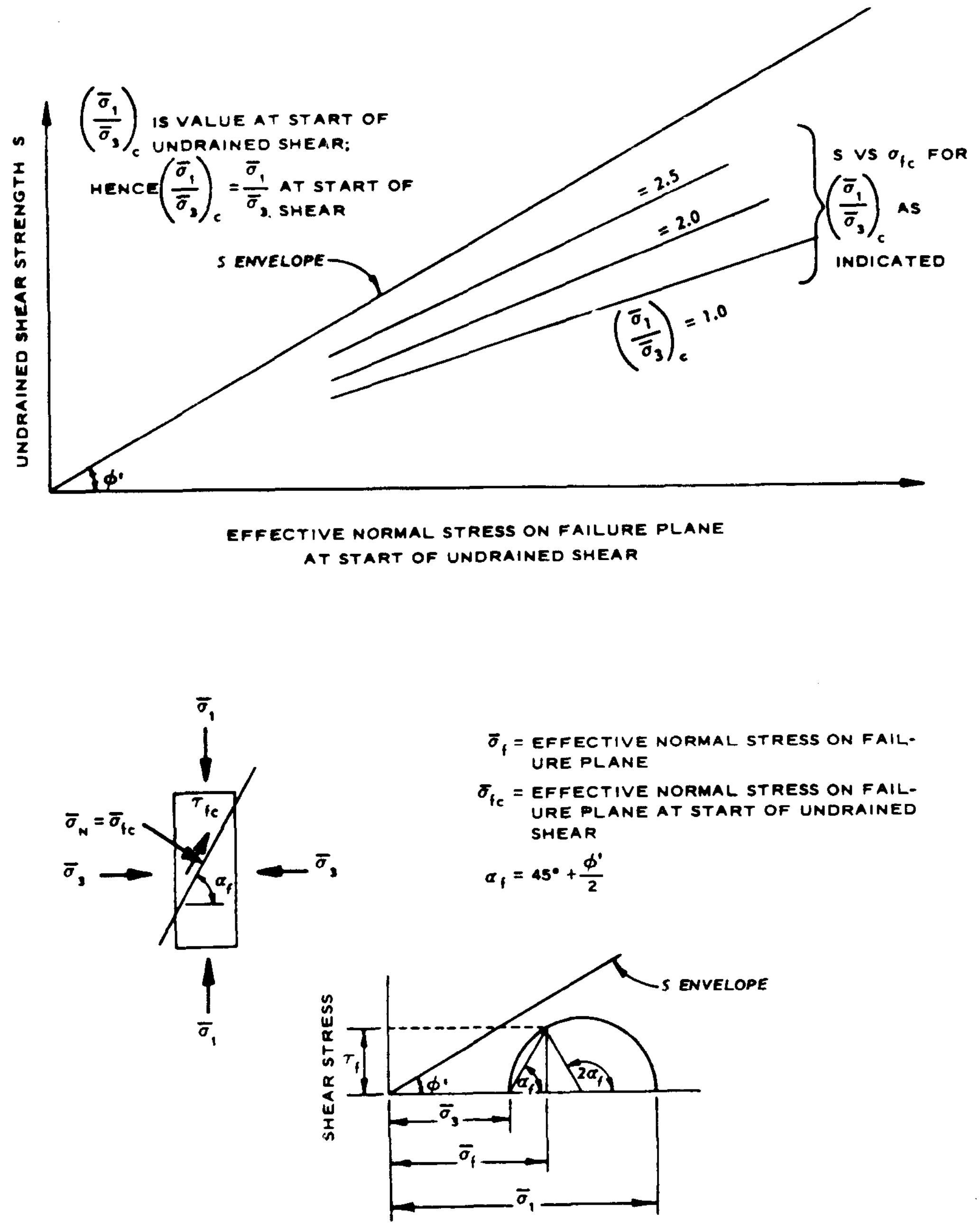
DATA FOR ESTIMATING PORE PRESSURES IN Q TEST

Plate VIII-6



UNDRAINED SHEAR STRENGTH
FOR VARIOUS RATIOS OF $\bar{\sigma}_1/\bar{\sigma}_3$
AT START OF SHEAR

Plate VIII-7



UNDRAINED SHEAR STRENGTH FOR FIELD STRESS CONDITIONS

☆U.S. GOVERNMENT PRINTING OFFICE: 1994 - 521-947/81206